Greg Gibson
Vice President, Regulatory Affairs

750 East Pratt Street, Suite 1600
Baltimore, Maryland 21202

UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

December 06, 2010

UN#10-302

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3,
RAI 268, Stability of Subsurface Materials and Foundations

Reference: James Steckel (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 268 RGS2 5120" email dated November 4, 2010

The purpose of this letter is to respond to the request for additional information (RAI) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated November 4, 2010 (Reference). This RAI addresses Stability of Subsurface Materials and Foundations, as discussed in Section 2.5.4 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 Combined License Application (COLA), Revision 6.

The enclosure provides our response to RAI 268, Question 02.05.04-24 and Question 02.05.04-25 and includes revised COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

Our response does not include any new regulatory commitments. This letter does not contain any sensitive or proprietary information.

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NRO

A response to Questions 02.05.04-26, 02.05.04-27, and 02.05.04-28 will be provided by January 31, 2011.

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Wayne A. Massie at (410) 470-5503.

*I declare under penalty of perjury that the foregoing is true and correct.*

Executed on December 06, 2010

Greg Gibson

Enclosure: Response to NRC Request for Additional Information RAI 268, Question 02.05.04-24 and Question 02.05.04-25, Stability of Subsurface Materials and Foundations, Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
Laura Quinn, NRC Environmental Project Manager, U.S. EPR COL Application
Getachew Tesfaye, NRC Project Manager, U.S. EPR DC Application (w/o enclosure)
Loren Plisco, Deputy Regional Administrator, NRC Region II (w/o enclosure)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2
U.S. NRC Region I Office

GTG/SJS/mdf

# Enclosure

## Response to NRC Request for Additional Information RAI 268, Question 02.05.04-24 and Question 02.05.04-25, Stability of Subsurface Materials and Foundations,

## Calvert Cliffs Nuclear Power Plant, Unit 3

**RAI 268**

**Question 02.05.04-24**

**Question 02.05.04-24** (determining the extent of the excavation)
In response to RAI Question 02.05.04-04, you stated that to confirm that the excavation has reached the load bearing Stratum IIb-Chesapeake Cemented Sand, two methods may be used: (1) proof rolling the entire excavated area until the grade offers a relatively unyielding surface; (2) dynamic cone penetration (DCP), and/or sand-cone in-situ compaction testing methods. Since the proof rolling method may not be a practical and reliable method to identify a specific soil layer, and since one of the indications of the Stratum IIb-Chesapeake Cemented Sand is that the SPT N-value is greater than 20 for the top layer of this stratum, please provide the following information:

1. Clearly specify which method(s) will be used during excavation to ensure the excavation reaches Stratum IIb; and

2. Provide the correlation between the SPT N-values and the values obtained from the field compaction test method (DCP and/or sand cone tests); or justify why the proposed field compaction tests can identify the Stratum IIb soil.

These clarifications will assist the staff determine if the proposed methods ensure that the excavation reaches the designed load bearing layer, thus meeting the design requirement and ensuring the stability of foundations in accordance with 10 CFR 100.23.

**Response**

Proof rolling and testing using ASTM D1556 "Test Method for Density and Unit Weight of Soil in-Place by Sand-Cone Method" as methods to confirm that the excavation has reached the load bearing Stratum IIB are removed from the FSAR. Dynamic Cone Penetration (DCP) testing by means of ASTM D7380-08 "Standard Test Method for Soil Compaction Determination (Dynamic Cone Penetration)" will be utilized due to its ease of use in the field. The existing in-situ materials that provide a competent foundation are defined as Stratum IIb-Chesapeake Sand. These materials are typically light to dark gray in color and have a SPT "N" value generally greater than 20. On average, the Stratum IIb – Chesapeake Cemented Sand is approximately 22.5 feet below the existing ground surface.

To confirm that the excavation has reached the load bearing Stratum IIB, the Geotechnical Engineer will develop a chart that provides a correlation between SPT N-values and the DCP values obtained from ASTM STP 399, "Dynamic Cone for Shallow In-Situ Penetration Testing." ASTM STP 399 provides a correlation between the DCP and SPT; however, using site specific information will increase the accuracy of the correlation. This chart will be developed with the SPT data that has been collected to date and correlated with DCP values after applying a correction for the overburden. Additional testing and correlation will be performed after excavation has begun and will be completed when the Stratum IIb-Chesapeake Cemented Sand is near the surface. In addition, once the Stratum IIb-Chesapeake Cemented Sand has been exposed, as identified by the Geotechnical Engineer, grain size analysis will be performed and the material will be photographed with appropriate color coding.

Once the design elevation is reached during excavation, DCP testing will be performed to characterize the subsurface conditions. In addition, samples will be collected for grain size analysis. The suitability of the design elevation will be determined based on DCP test correlation, grain size, and the soil color code. The grain size and soil color will help differentiate between Stratum IIa - Chesapeake Clay/Silt and Stratum IIb - Chesapeake Cemented Sand.

Structural backfill placement will not begin until the unsuitable material of the final excavation grade has been verified and approval received from the Geotechnical Engineer. The Geotechnical Engineer will be responsible for final approval of the foundation soils. A geologist will map the exposed stratum. Photos and videotape of the exposed stratum will be collected for documentation. Finally, acceptance will be documented on a Final Foundation Acceptance form that is completed by the responsible parties and included in the report.

**COLA Impact**

The text inserted into FSAR Section 2.5.4.5.2 in response to RAI Question 02.05.04-04[1], will be replaced as shown:

> ~~Once the design foundation elevation is reached, two methods of verifying the competent foundation soils may be used. The first method is to proof roll the entire excavated area with a compaction vehicle or approved equivalent until the grade offers a relatively unyielding surface (i.e., less than one inch). Any areas that exhibit excessive (i.e., greater than one inch) rutting, pumping or yielding will be identified by the Geotechnical Engineer and the construction contractor will undercut these areas until the intended competent Stratum is encountered as verified by additional proof rolling. The second method is to perform in-situ compaction testing by means of ASTM D7380-08 "Standard Test Method for Soil Compaction Determination (Dynamic Cone Penetration)" (ASTM 2008b) and/or ASTM D1556 "Test Method for Density and Unit Weight of Soil in-Place by Sand-Cone Method" (ASTM 2007b). Structural backfill placement will not begin until the unsuitable material of the final excavation grade has been verified and approval received from the Geotechnical Engineer.~~
>
> <u>To confirm that the excavation has reached the load bearing Stratum IIB, the Geotechnical Engineer will develop a chart that provides a correlation between SPT N-values and the Dynamic Cone Penetration (DCP) values obtained from ASTM STP 399 (ASTM, 1966). ASTM STP 399 provides a correlation between the DCP and SPT; however, using site specific information will increase the accuracy of the correlation. This chart will be developed with the SPT data that has been collected to date and correlated with DCP values after applying a correction for the overburden. Additional testing and correlation will be performed after excavation has begun and will be completed when the Stratum IIb-Chesapeake Cemented Sand is near the surface. In addition, once Stratum IIb-Chesapeake Cemented Sand has been exposed, as identified by the Geotechnical Engineer, grain size analysis will be performed and the material will be photographed with appropriate color coding.</u>

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[1] G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 218, Stability of Subsurface Materials and Foundations," Letter UN#10-105, dated April 7, 2010.

DCP testing by means of ASTM D7380-08 (ASTM, 2008b) will be utilized due to its ease of use in the field. Once the design elevation is reached during excavation, DCP testing will be performed to characterize the subsurface conditions. In addition, samples will be collected for grain size analysis. The suitability of the design elevation will be determined based on DCP test correlation, grain size, and the soil color code. The grain size and soil color will help differentiate between Stratum IIa - Chesapeake Clay/Silt and Stratum IIb - Chesapeake Sand.

Structural backfill placement will not begin until the unsuitable material of the final excavation grade has been verified and approval received from the Geotechnical Engineer. The Geotechnical Engineer will be responsible for final approval of the foundation soils. A geologist will map the exposed stratum. Photos and videotape of the exposed stratum will be collected for documentation. Finally, acceptance will be documented on a Final Foundation Acceptance form that is completed by the responsible parties and included in the report.

The following reference will be added to FSAR Section 2.5.4.13:

**ASTM, 1966.** ASTM STP 399, "Dynamic Cone for Shallow In-Situ Penetration Testing." American Society for Testing and Materials (ASTM), 1966.

**RAI 268**

**Question 02.05.04-25**

**02.05.04-25** (Bearing capacity calculation)
In response to RAI Question 02.05.04-15, you described three methods used in a static bearing capacity sensitivity analysis and compared the analysis results. In order for the staff to complete a detailed review and to ensure the stability of foundations in accordance with 10 CFR 100.23, additional details and explanation are needed. Specifically provide the following:

1. Describe how the foundation dimensions were determined and used as input in the Slope/W and Plaxis 2D analyses. Also describe if non-uniform loading condition(s) on the foundation were considered.

2. Figures 6 and 7 in the RAI response present the ultimate bearing capacity analysis results, and you stated that the ultimate bearing capacity was reached when "a significant decrease in stiffness was observed" during the Plaxis 2D model analysis. Describe and justify the criterion that was used to determine the stiffness that corresponds to the ultimate bearing capacity.

**Response**

1. The bearing capacity analysis outlined in the response to RAI Question 02.05.04-15[2] was based on an equivalent rectangular shape representing the Nuclear Island (NI). The equivalent shape has the same moment of inertia as the original footshape. The equivalent foundation width and length were B = 270 ft and L = 300 ft, respectively. The non-uniformity of loads on the NI was evaluated by calculating the moment at the center of the footprint. The moment due to the loads of each building was calculated using the equivalent point load representing the area load. The moment arms were in accordance with Figure 1. The equivalent loads, moment arms, and moments for each building at the NI are given in Table 1.

   Effective equivalent width and length due to the eccentricity ($e_B$ and $e_L$ in both directions) created by the moment at the center of the NI are $B' = B - 2e_B = 268.9$ ft and $L' = L - 2e_L = 298.1$ ft. Effective equivalent width and length are very close to their equivalent counterparts, which formed the basis for the assumptions of using average load over the NI, and using equivalent width in Plaxis 2D and Slope/W models.

2 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 218 and RAI 229, Stability of Subsurface Materials and Foundations," Letter UN#10-207, dated July 23, 2010.

**Figure 1**
**Center Points for the Nuclear Island Buildings**

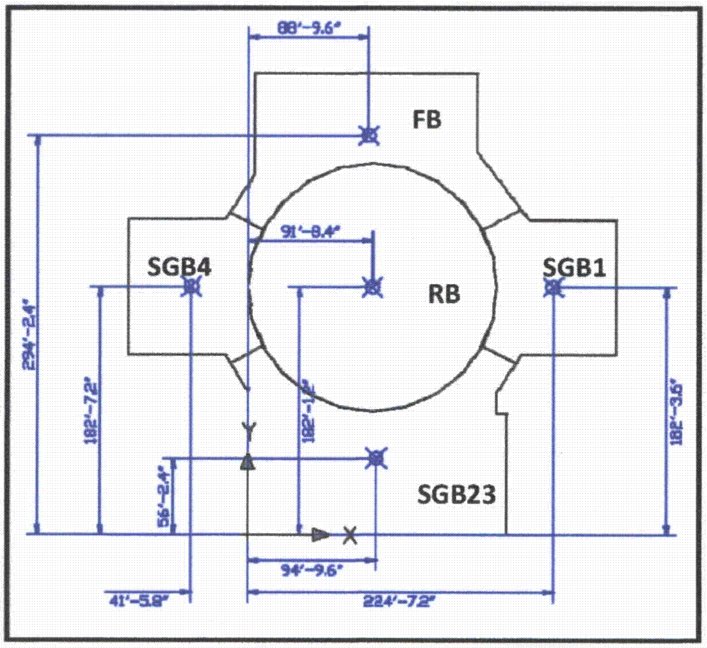



**Table 1**
**Equivalent Loads and Areas for the Nuclear Island Buildings**

| BUILDING | | P (KIPS) | A ($Ft^2$) | W (KSF) | XC[1] (Ft) | YC[1] (Ft) |
|---|---|---|---|---|---|---|
| RB | Reactor Building | 313477 | 26268 | 11.9 | 91.7 | 182.1 |
| FB | Fuel Building | 216806 | 14545 | 14.9 | 88.8 | 294.2 |
| SGB1 | Safeguard 1 | 108064 | 9198 | 11.7 | 224.6 | 182.3 |
| SGB23 | Safeguard 2&3 | 200814 | 20952 | 9.6 | 94.8 | 56.2 |
| SGB4 | Safeguard 4 | 104079 | 9247 | 11.3 | -41.5 | 182.6 |

(1) Coordinates according to the origin shown in Figure 1.

2. The ultimate bearing capacity is determined based mainly on the change in the slope of the load-deformation curve obtained from Plaxis 2D.

   The Vesic method and Slope/W analysis result in an ultimate load that initiates failure. Plaxis 2D provides load deformation curves, where the failure point needs to be defined along the load-deformation curve. Thus, obtaining the bearing capacity from the finite element software Plaxis 2D implies integration of the stability and serviceability criteria, and, when used to check stability, it is only used for checking the range of bearing capacities obtained from Slope/W analysis and hand calculations accordingly. Furthermore, the factor of safety for bearing capacity type of failure was greater than 22 for the average parameters, and greater than 8 for the lower bound parameters, considering the average Nuclear Island pressure of 11.8 ksf. The factors of safety are much larger than the minimum required factor of safety of 3. The justification for the determination of a load level that triggers a significant softening in Plaxis 2D analysis is as follows:

   The load-deformation curves obtained from Plaxis 2D for average and lower bound parameters are shown in Figures 2a and 2b, respectively. For both cases, the initial linear elastic response gradually shifts into the non-linear response. Following the end of the initial linear elastic section (Point A) the stiffness of the soil medium underneath the foundation starts decreasing. This reduction represents the redistribution of stresses from the overstressed (plastic) soil elements to the soil elements that are still experiencing linear elastic behavior. Initially, the plastic soil elements are at the corners of the foundation. Then, the accumulation of plastic points starts forming the expected bearing capacity failure pattern. The load-displacement curves shown in Figures 2a and 2b are divided into 5 sections. The first sections (Line 1) in each figure represent the initial linear elastic behavior. The total deformation contours at the end of initial linear elastic zone are shown in Figure 3a for average parameters and Figure 4a for lower bound parameters. Typical bearing capacity type failure is not yet formed at the end of initial linear elastic zone (Point A in Figures 2a and 2b). Expected bearing capacity type failure looks more like deformation profiles shown in Figure 3b for average parameters and Figure 4b for lower bound parameters. These deformation profiles are observed at Point B in Figures 2a and 2b.

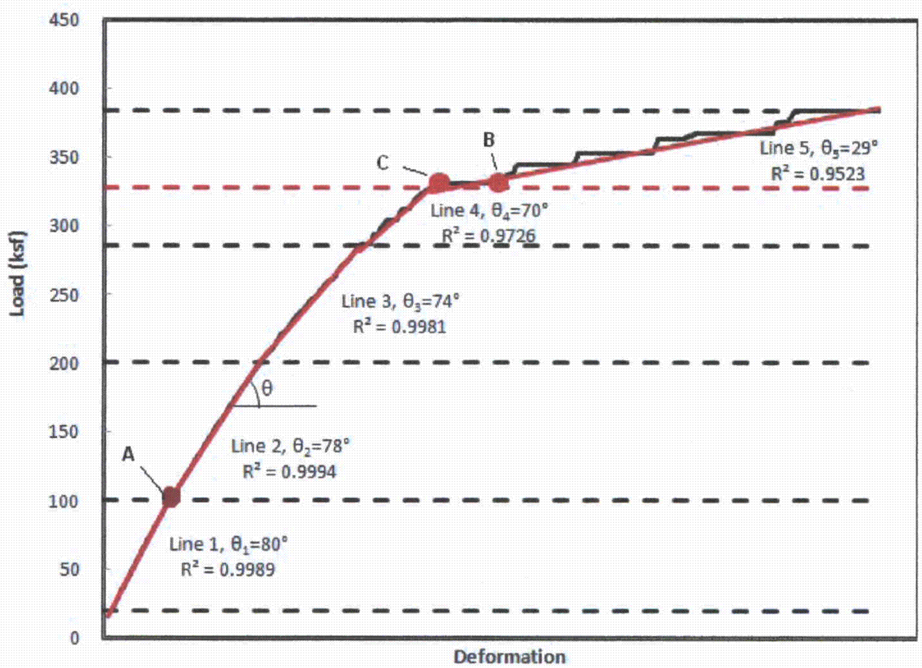
   The slope of the load-deformation curve (stiffness) decreases more than 50% at the load levels identified as bearing capacity in RAI Response 02.05.04-15[2] (Point C in Figures 2a and 2b). The bearing capacity obtained by examining the slope of the slope load-deformation curve at Point C is the same as the one obtained looking at the deformation profiles for average parameters (at Point B in Figure 2a), and slightly below the one obtained looking at the deformation profiles for lower bound parameters (at Point B in Figure 2b). Therefore, the ultimate bearing capacity is assumed to be at Point C in Figures 2a and 2b. Consequently, the bearing capacity obtained from load-deformation profile is not based on a predetermined specific stiffness, but based on an evaluation of load-deformation profile from the perspective of expected failure patterns and associated significant reduction in stiffness (not the instantaneous value). Furthermore, the bearing capacities obtained are in agreement with the Vesic method and the Slope/W analysis.

### COLA Impact

The COLA will not be revised as a result of this response.

**Figure 2**
**Load Deformation Curves**

## a. Average Parameters




## b. Lower Bound Parameters

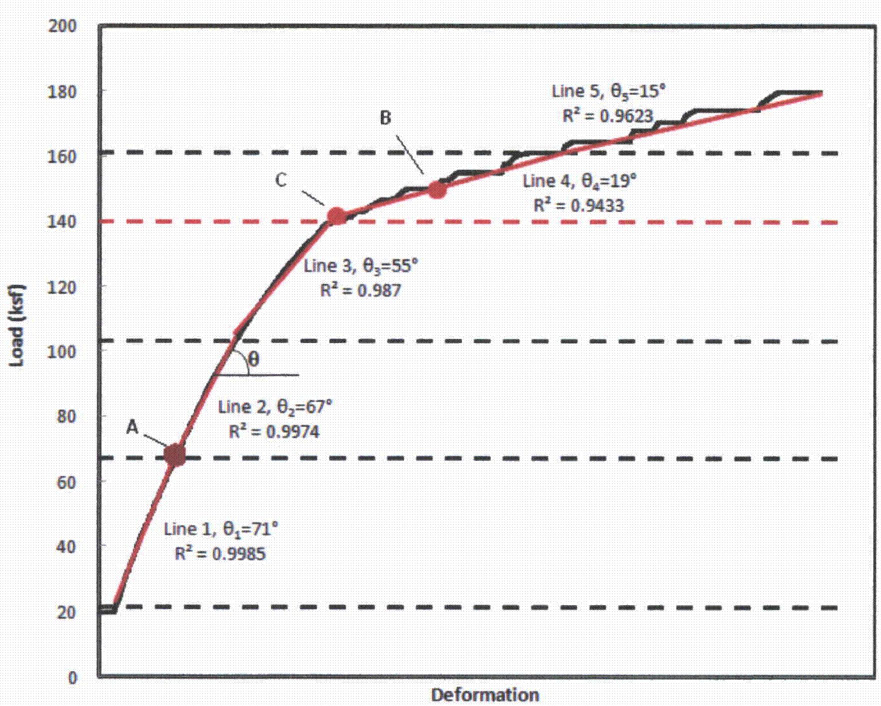
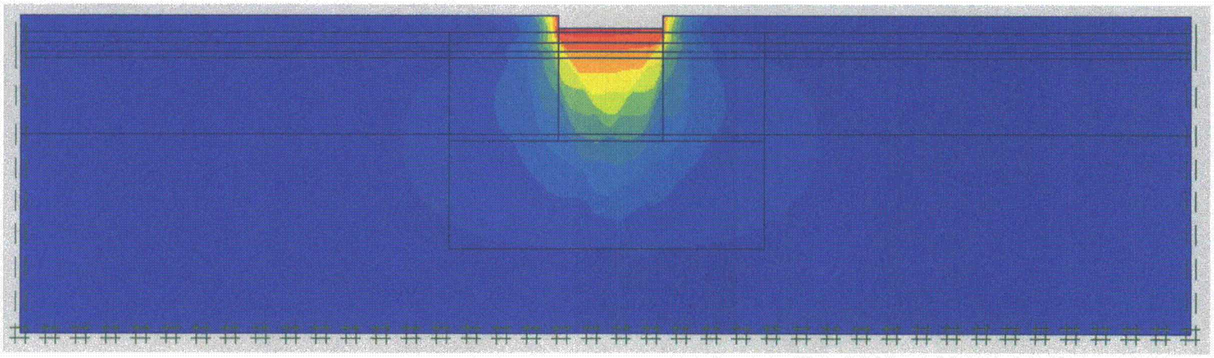


**Figure 3**
**Total Deformation for Average Parameters (Figure 2a)**

**a. At Point A, the end of the Initial Linear Elastic Response**



**b. At Point B, the Load Expected to Produce a Bearing Capacity Type of Failure**

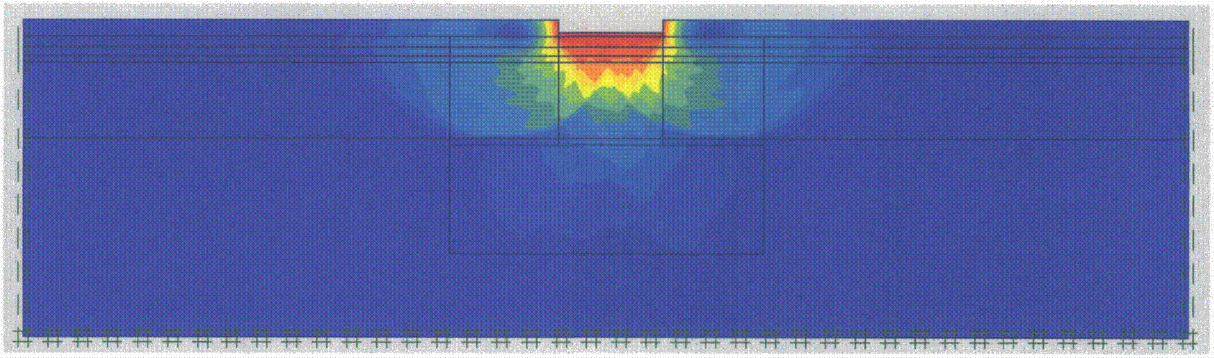
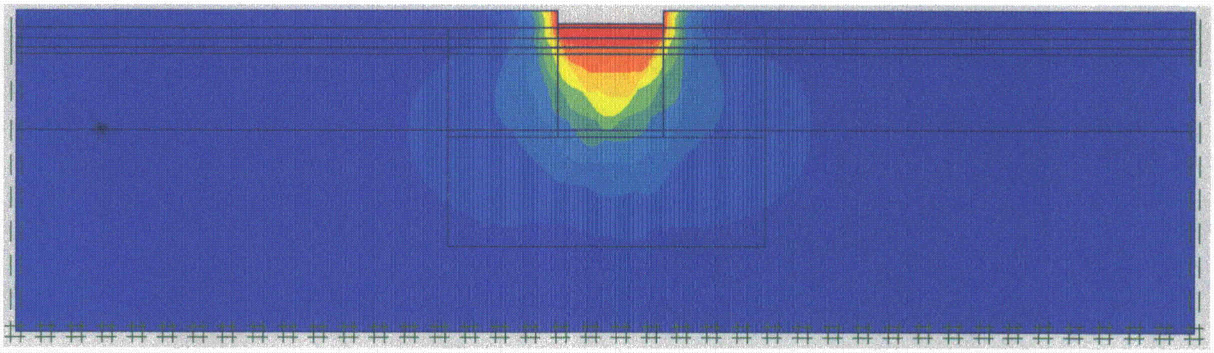


**Figure 4**
**Total Deformation for Lower Bound Parameters (Figure 2b)**

**a. At Point A, the end of the Initial Linear Elastic Response**



**b. At Point B, the Load Expected to Produce a Bearing Capacity Type of Failure**